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OPEN-FILE REPORT 3-BAS

METHODOLOGY AND RESULTS OF IN-SITU PERMEABILITY TESTS IN UNWEATHERED GLACIAL TILL OF SOUTH DAKOTA

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CONTENTS

FIGURES

Page

TABLES

ABSTRACT

The South Dakota Geological Survey has installed a network of piezometers in glacial till sediments of eastern South Dakota. In-situ permeability tests were conducted on 30 of these piezometers completed in unweathered glacial till. Two methods were used to analyze the data. These were the Hvorslev time-lag method and the Luthin and Kirkham piezometer test method. Results from the Hvorslev method showed an average hydraulic conductivity in the unweathered till of 2.0×10^{-8} cm/s. Results from the Luthin and Kirkham method showed an average hydraulic conductivity in the unweathered till of 2.6×10^{-8} cm/s. These hydraulic conductivity values demonstrate the extremely low permeability of unweathered glacial till.

INTRODUCTION

The region of South Dakota east of the Missouri River is covered mostly by glacial deposits. A large fraction of these glacial deposits is till, a poorly sorted material with a clay content generally in excess of 20 percent. A much debated question of scientific and economic importance concerns the mechanism of ground water movement in glacial tills. One of the properties of glacial till which must be known before this question can be answered is its hydraulic conductivity.

Barari (1983) presented discussion suggesting that movement of ground water through unweathered glacial till is insignificant or nonexistent. Since then, the South Dakota Geological Survey has installed a network of approximately 200 observation wells and piezometers in glacial till in eastern South Dakota for the purpose of gathering data (chemical and hydraulic) to define the hydrology of glacial till.

Three study areas were chosen, each of which has unique characteristics. The CENDAK area was chosen because of interest in the development of large scale irrigation there. The Dolton area was chosen because a rural water system there began pumping ground water in 1982 from a confined outwash aquifer with no known surficial connection. In 1985 a second rural water system began pumping out of the same aquifer. The Lincoln County area was chosen because it is a glacial till area underlain by an outwash which is under confined, flowing conditions. Figure 1 depicts the general locations for the observation well and piezometer network. Barari and Hedges (1985) presented early data from two of these three areas. Cravens (1985) also presented data from the CENDAK area. This report presents comprehensive data on the permeability of unweathered till from all three study areas.

Glacial till deposits consist of two units, a weathered unit and an unweathered unit. The weathered unit overlies the unweathered unit and is generally 20 to 40 feet thick. The weathered till zone has permeability values two to three orders of magnitude greater than unweathered till due to the fractures and macropores present in the weathered till (Prudic, 1982). The results reported in this paper are for in-situ permeability tests conducted in unweathered till. As a result of its mechanism of deposition, unweathered till is probably characterized by isotropic permeability. That is, the horizontal and vertical permeabilities are very similar. Therefore, the results are reported simply as hydraulic conductivity with no distinction made between vertical or horizontal hydraulic conductivity.

TEST METHODS

Piezometer Construction

The two types of piezometer construction used for this permeability testing were screened intake and cored intake.

Construction of a screened-intake piezometer is as follows:

1. Auger 4-inch diameter hole to desired depth.
2. Insert 2-inch diameter schedule 80 PVC casing with 2-foot long slotted screen in auger hole.
3. Add gravel pack to cover the screen.
4. Add granular bentonite to within 2 feet of land surface.
5. Fill the rest of the annulus with native material.

Figure 2 depicts the construction method for a screened-intake piezometer.

Construction of a cored-intake piezometer is as follows:

1. Auger 4-inch diameter hole to desired depth.
2. Insert 2-inch diameter schedule 80 PVC casing (with sharpened end of casing covered by a thin plastic cap) into auger hole.
3. Push casing approximately 3 inches into bottom of hole (thereby cutting through thin plastic cap).
4. Core bottom-hole sediment through inside of casing an additional 12 inches with a 1.5-inch diameter thin-wall sampler (Shelby tube).
5. Pump bentonite slurry into annulus from bottom up to within 2 feet of land surface.
6. Fill the rest of the annulus with native material.

Figure 3 depicts the construction method for a cored-intake piezometer.

Major deviations (e.g., different piezometer intake dimensions) from the general construction procedures outlined above will be indicated in the well log for that particular piezometer. Well logs are on file at the South Dakota Geological Survey and are available on request.

Advantages and Disadvantages of Cored-Intake Piezometers

The advantages of a cored-intake piezometer are that it has no screen which may become partially or fully plugged with fine particles, thereby indicating a permeability less than that of the medium itself. A cored-intake piezometer forms a natural seal out of the native material between the intake area and the bentonite in the annulus. This seal should effectively eliminate error in permeability measurements which could result from seepage of fluid through the bentonite into the piezometer intake. This error component may be significant in low permeable materials where the hydraulic conductivity of bentonite may be higher than that of the native material. Data from the American Colloid Company (Terry, undated) show the hydraulic conductivity of a bentonite grout to range from 1.0×10^{-8} centimeters per second (cm/s) to 1.0×10^{-7} cm/s.

Disadvantages of a cored intake are that the cored area relies on the stability of the native material for support. This stability is usually sufficient to hold the intake open in clayey materials. In some cases, however, the cored area has partially collapsed (indicated by a decreasing piezometer intake area). This makes it difficult to determine what the actual intake dimensions are and introduces some error in permeability calculations. The magnitude of this error, however, is generally less than one-third of an order of magnitude and is a relatively small error. In materials with low permeability, the length of time required to conduct a permeability test can range from several months to more than a year due to the rates of low volume transfer. The smaller intake area of the cored-intake piezometers is a factor contributing to an increased testing time. This is due to the fact that volume-transfer rates from the medium tested into the piezometer are proportional to the piezometer intake area.

Data Acquisition

Water level measurements were taken as water levels in the piezometers recovered from some point below the static water level. Some of the measurements began from the time the piezometer was installed. In other piezometers, the measurements began after water had been evacuated from the piezometers with a bailer.

Data Analysis

A water level hydrograph was constructed for each piezometer. The hydrograph was constructed by plotting depth to water from casing top versus time. Figure 4 shows a representative hydrograph. All five piezometers (A, B, C, D, and E) at site GT-TU-2 are plotted on the graph in figure 4. Following the letter identifying each piezometer recovery curve on figure 4 is a number in parentheses. This number indicates piezometer depth in feet from the casing top. Piezometers A and B in figure 4 are completed in weathered glacial till and exhibit rapid response to precipitation. Piezometers C and D in figure 4 are completed in unweathered glacial till and exhibit typical logarithmic recovery curves for a low-permeability material. The unweathered glacial till piezometers show no response to precipitation. Piezometer E is completed in a bedrock aquifer (Niobrara Formation) directly underlying the unweathered glacial till. The water level hydrograph is useful in determining the validity of the data and piezometer function. Large deviations from the logarithmic recovery curve shape are indicative of invalid data or a failing piezometer (e.g., plugged intake, improper bentonite seal, or broken casing).

Two methods of data analysis were used to obtain hydraulic conductivity values. The method developed by Luthin and Kirkham (1949) and the time-lag method developed by Hvorslev (1951) were employed. The two methods use different approaches in the compilation of the data. A favorable comparison of hydraulic conductivity values obtained by the two methods should be indicative of valid data and piezometer function.

Luthin and Kirkham Method

The data analysis method developed by Luthin and Kirkham is based on the calculation of a permeability coefficient between every two successive data points. If a number, (n), data points are used, then n-1 permeability coefficients will be calculated. These values are then averaged to compute an overall hydraulic conductivity for the medium. Each permeability coefficient is based on the time lapse that occurs between two successive water level measurements. The piezometer intake area and the casing diameter are also taken into account.

Figure 5 shows the variables and formula used in the Luthin and Kirkham method. Luthin and Kirkham use a variable called the A-function in the denominator of their equation to account for the flow character of the piezometer intake. They experimentally determined the A-function for an intake 4 inches long at various intake diameters (fig. 6a). They also determined the A-function for an intake 1 inch in diameter at various intake lengths (fig. 6b). Since the A-function is proportional to the dimensions of the intake, the A-function for other intake dimensions can be calculated from figure 6. This is shown in the following example: The A-function for an intake 6 inches long and 2 inches in diameter is desired. An intake 4 inches long to be in the same proportion must have a diameter of 1.33 inches. The value of the A-function for an intake 4 inches long and 1.33 inches in diameter is read from figure 6a to be 14.0 inches. The value for the intake 6 inches long and 2 inches in diameter is then (6/4) x 14.0 inches = 21.0 inches.

Utilizing figures 6a and 6b, a graph of A/D versus W/D, where W = intake length, D = intake diameter, and A = A-function, was constructed (fig. 7). The data range of W/D derived from figures 6a and 6b is 0 to 8. Therefore, the curvilinear graph line in figure 7 was extended beyond a W/D value of 8 on the assumption that the A-function is continuous. From this extension, additional data points were obtained up to a W/D value of 20. The x,y pairs in the W/D range of 0 to 20 were then entered into a curve fitting program. This program then calculates an equation and its coefficients which best fit the data points. The output from this program is shown in figure 8. A sixth degree polynomial equation was found to fit the data well. W/D values in the range of 0 to 20 can be entered into the equation to calculate the proper A/D value (multiplication of A/D by D will yield the A-function). Any attempt to calculate an A/D value from a W/D value greater than 20 will result in gross error due to inflection points on the fitted curve beyond a W/D value of 20. That is, the equation may be used for interpolation, but not for extrapolation.

Hvorslev Method

The second data analysis method used is that of the time-lag theory developed by Hvorslev. The Hvorslev method analysis assumes a homogeneous, isotropic, infinite medium in which both soil and water are incompressible. Figure 9 depicts the variables, graph, and equation used in the Hvorslev analysis. The Hvorslev calculation is based on the principle that the rate of inflow, q, at the piezometer intake at any time, t, is proportional to the hydraulic conductivity, K, of the medium and to the unrecovered head difference. The Hvorslev equation also accounts for the dimensions of the intake and the casing diameter. If the ratio of unrecovered head to initial head is plotted on a logarithmic scale versus time as shown on the graph in figure 9, a linear function becomes evident. From this regression line, the basic time lag, T(o), used in the Hvorslev equation can be interpolated. T(o) is the time value interpolated from the regression line when the ratio of unrecovered head to initial head is 0.37. If the data from the test are reliable, the semi-log graph will show a correlation coefficient very near -1.0 and a y-intercept very near 1.0. This same type of graph can be constructed by plotting the common logarithm of the ratio of unrecovered head to initial head on a linear scale versus time. In this graph the y-intercept should be very near zero and the correlation coefficient should be very near -1.0.

TEST RESULTS AND DISCUSSION

A summary of hydraulic conductivities obtained in the analyses is given in table 1. The average hydraulic conductivities obtained from the two methods used agree very well (2.6×10^{-8} cm/s for the Luthin/Kirkham method and 2.0×10^{-8} cm/s for the Hvorslev method).

From the information in table 2, it can be seen that the hydraulic conductivity values of unweathered glacial till place it on the lower end of the clay range.

TABLE 1. Summary of permeability test results

NUMBER OF PIEZOMETERS TESTED: 30

(CORED INTAKE: 27)
(SCREENED INTAKE: 3)

MINIMUM PIEZOMETER DEPTH: 18.2 ft. (from casing top)
MAXIMUM PIEZOMETER DEPTH: 102.6 ft. (from casing top)

METHOD	AVERAGE K	MINIMUM K	MAXIMUM K
LUTHIN/KIRKHAM	2.6×10^{-8} cm/s	6.2×10^{-9} cm/s	7.2×10^{-8} cm/s
HVORSLEV	2.0×10^{-8} cm/s	7.5×10^{-9} cm/s	4.7×10^{-8} cm/s

TABLE 2. Ranges of hydraulic conductivities for unconsolidated sediments
(modified from Fetter, 1980, p. 75)

Material	Conductivity (cm/s)
Clay	1.0×10^{-9} to 1.0×10^{-6}
Silt, clayey sands	1.0×10^{-6} to 1.0×10^{-4}
Fine sands	1.0×10^{-5} to 1.0×10^{-3}
Glacial outwash	1.0×10^{-3} to 1.0×10^{-1}
Well-sorted gravel	1.0×10^{-2} to 1

A more practical perspective of what these permeabilities indicate can be obtained if Darcy's law is employed to calculate a flow velocity. The Hvorslev average, 2.0×10^{-8} cm/s converts to 0.25 inches/year. Typical vertical gradients in unweathered till are on the order of 0.4. Darcy's law holds that the velocity of fluid movement through a porous medium is equivalent to the hydraulic gradient multiplied by the hydraulic conductivity for the fluid/porous medium system, ($v=Ki$). For simplicity, the negative sign in the equation has been ignored. Employing this relationship for unweathered till we obtain:

$$\text{Darcy velocity} = (0.25 \text{ in/yr}) \times (0.4) = 0.10 \text{ in/yr}$$

This calculation is based on the assumption that Darcy's law is valid and linear, even in low-permeability materials. If this is the case, a graph of velocity versus gradient will yield a linear relationship with an x-intercept of zero (fig. 10, solid line). In fact, it may be that the law does not completely hold for low-permeability materials (Swartzendruber, 1962 and Li, 1963). In materials such as shales and unweathered glacial till, a threshold hydraulic gradient may have to be exceeded before flow begins. In this case, a graph of velocity versus gradient will generate the relationship depicted in figure 10 (dashed line). The threshold hydraulic gradient is given by the x-intercept, $i(t)$. It is recognized that the Darcy velocity (discharge velocity) is different than the seepage velocity. However, in determining the amount of water that can flow through a porous medium, the Darcy velocity must be used.

If the concept of a threshold hydraulic gradient is valid for unweathered glacial till, the discharge velocity will be less than 0.10 in/yr and may even be equal to zero, depending on the magnitude of the threshold gradient. In either case, the permeabilities of unweathered glacial till reported here support the theory by Barari and Hedges (1985) that water movement through this hydrologic unit is insignificant or nonexistent.

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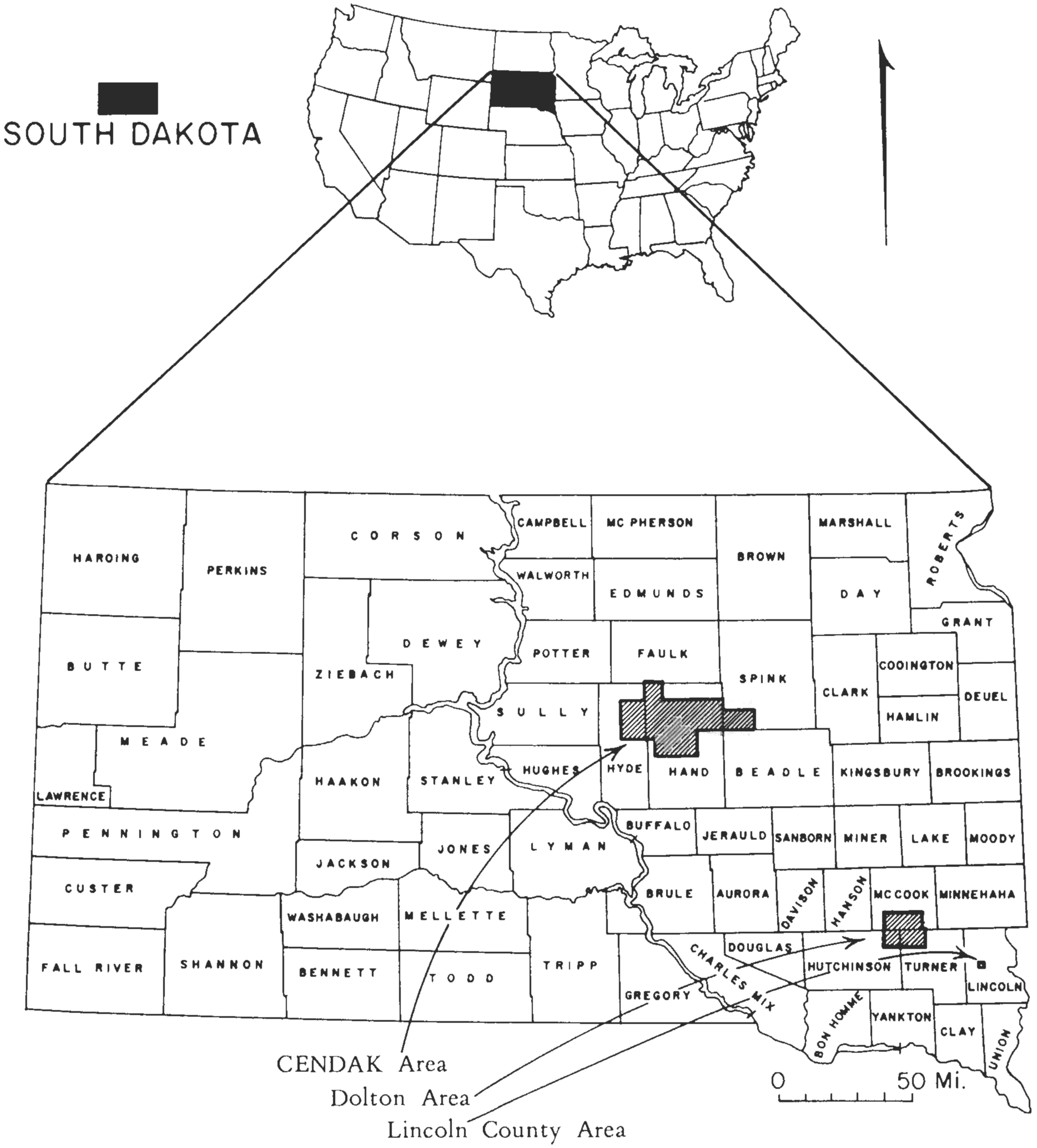


FIGURE 1. Locations of glacial till research areas

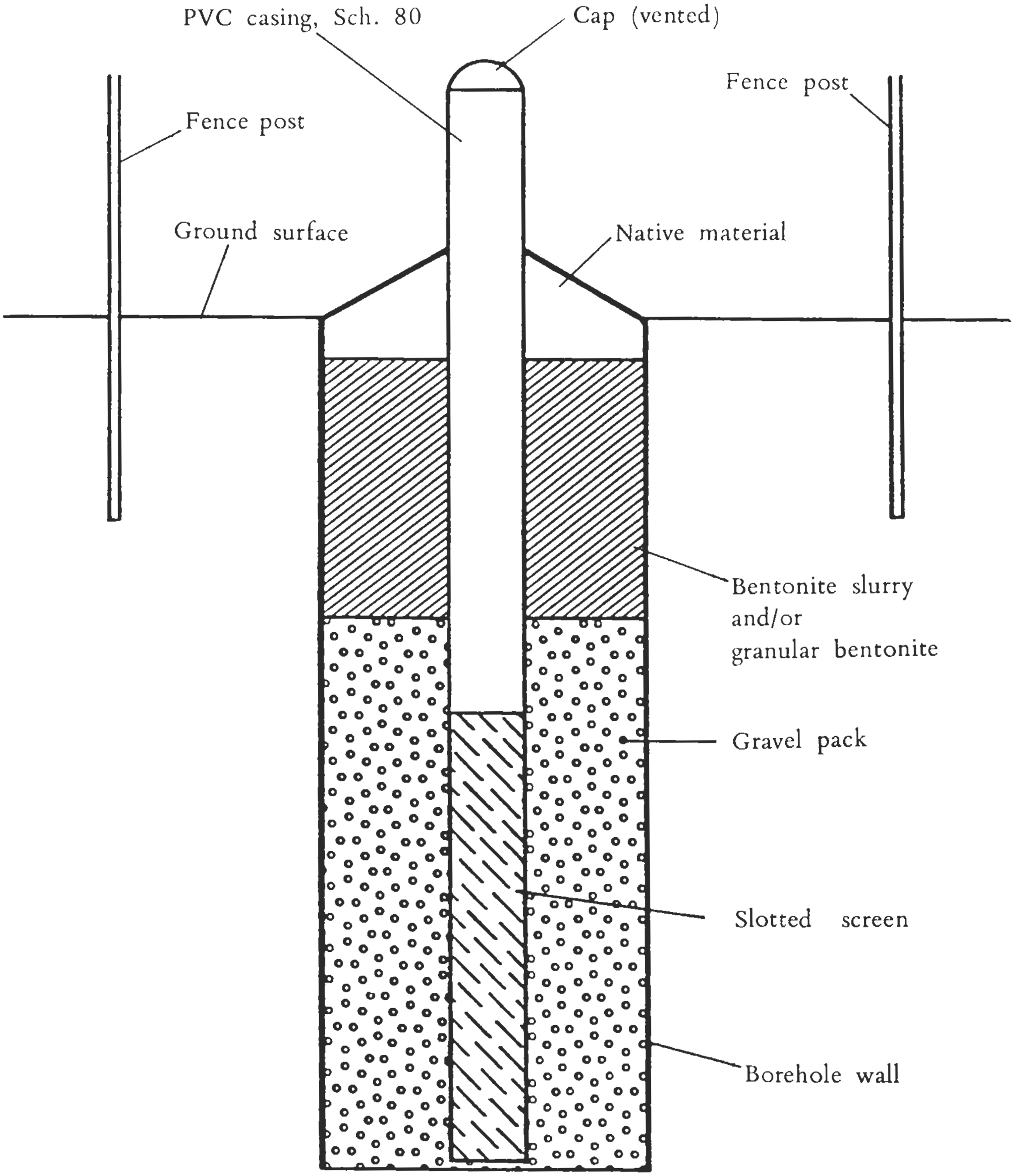


FIGURE 2. Screened intake piezometer construction (not to scale)

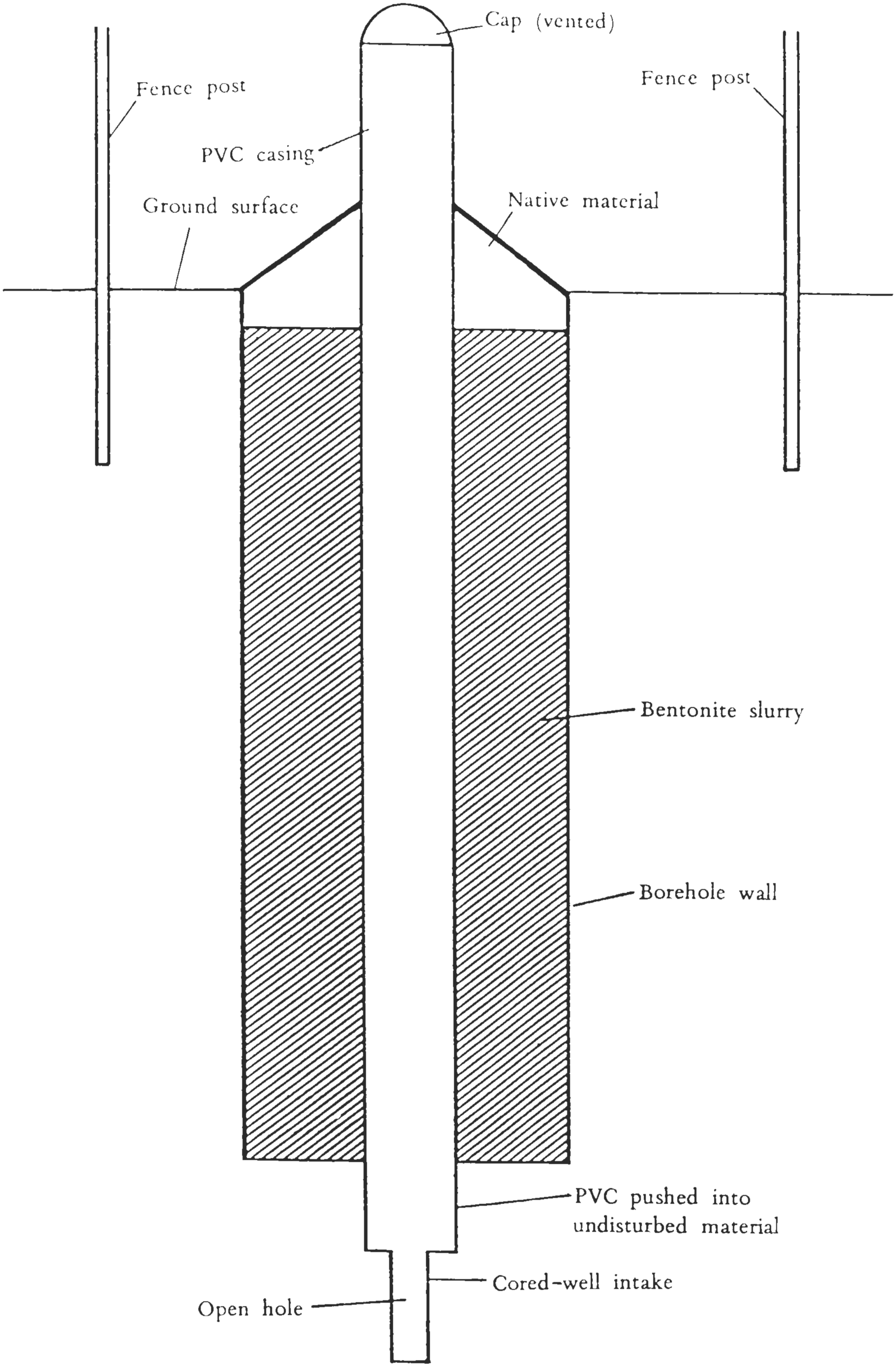


FIGURE 3. Cored intake piezometer construction (not to scale)

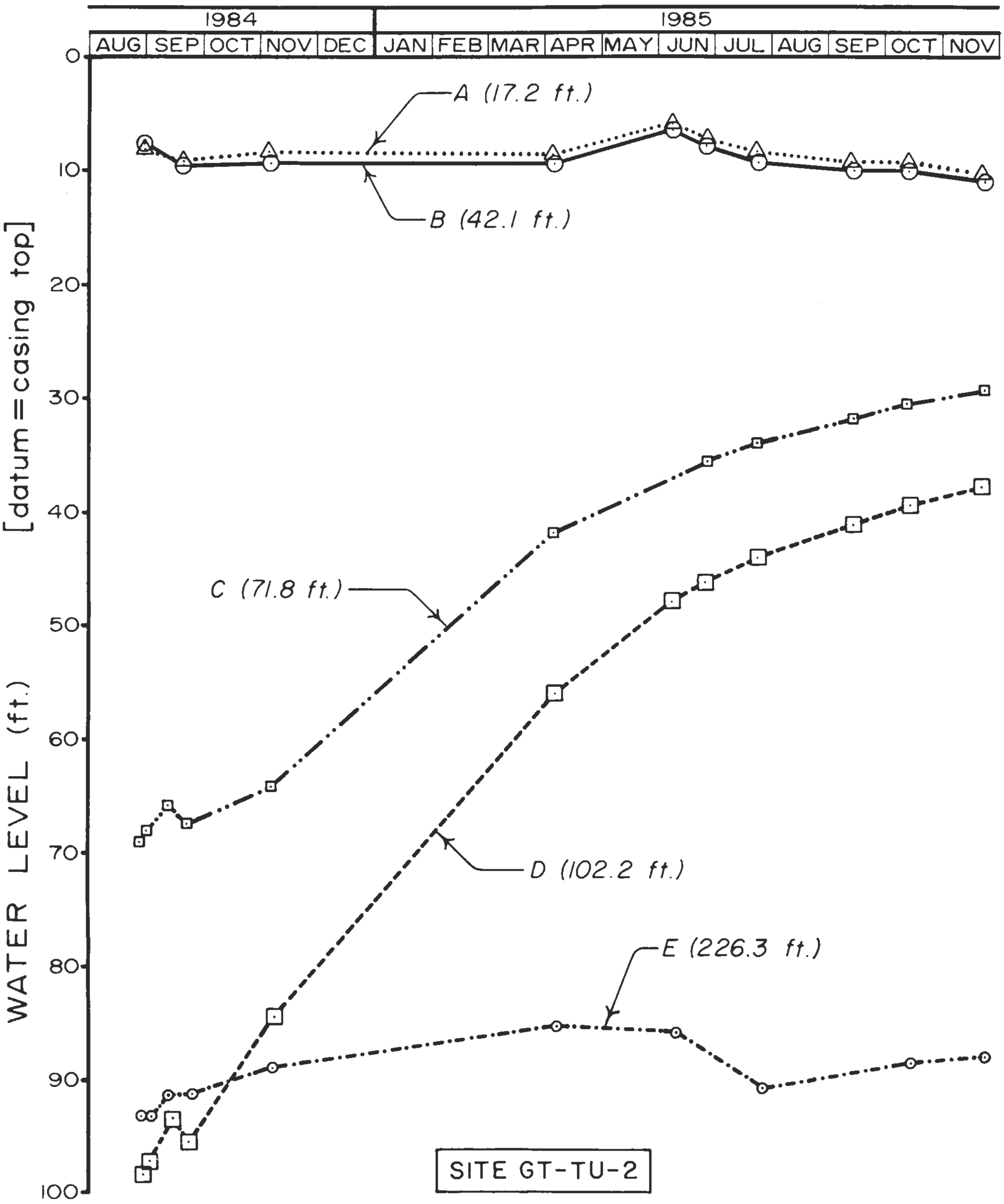
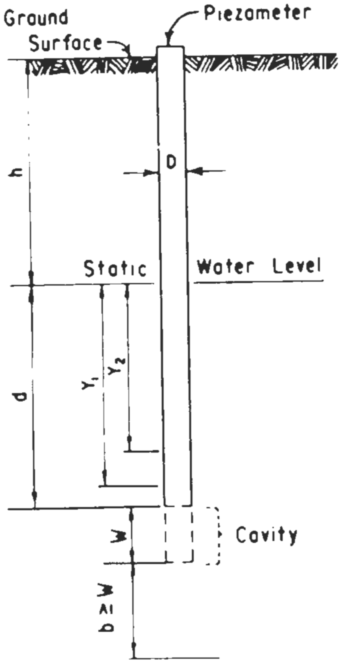


FIGURE 4. Water level hydrograph

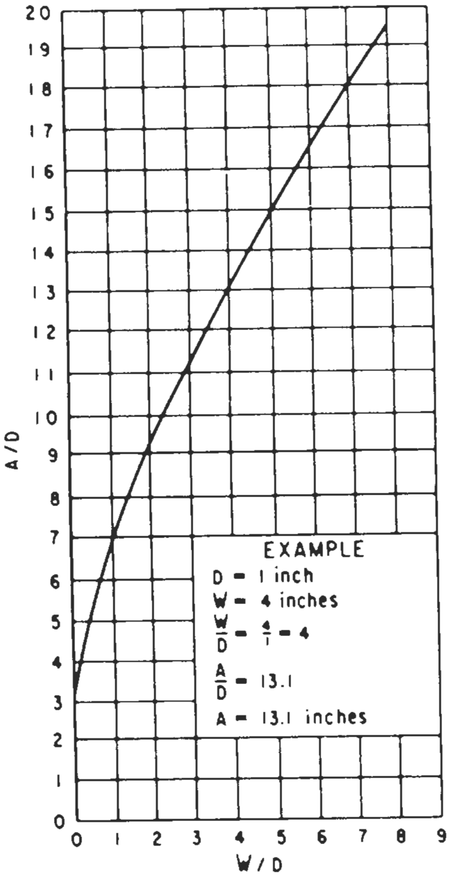
Location : Hole C-2 -- Sample Farm
Observer : A.P.B. Date : October 9, 1974



h = 86.40 Ground surface to static water level (inches)
D = 1.0 Inside diameter piezometer and cavity (inches)
d = 93.60 Static water level to bottom of piezometer (inches)
W = 4.0 Length of cavity (inches)
A = 13.1 Constant for a given flow geometry taken from curve (inches)
K = Hydraulic conductivity (inches per hour)
b = Depth to texture change
Y_1, Y_2 = Distances from static water level to water level at times t_1 and t_2 (inches)
$(t_2 - t_1)$ = Time for water level to change from Y_1 to Y_2 (seconds)

$$K = \frac{3{,}600\ \pi\ (D/2)^2 \log_e (Y_1 / Y_2)}{A\ (t_2 - t_1)} \text{(inches per hour)}$$

Time (seconds)		Y (inches)		A	$t_2 - t_1$	Y_1/Y_2	$\log_e$ Y_1/Y_2	$3{,}600\ \pi$ $x\ (D/2)^2$	K
initial (t_1)	Final(t_2)	Initial(Y_1)	Final(Y_2)						
0	30	86.00	77.90	13.1	30	1.104	0.099	2827.44	0.71
30	60	77.90	70.25	13.1	30	1.109	0.104	2827.44	0.74
60	90	70.25	63.00	13.1	30	1.115	0.107	2827.44	0.78
90	120	63.00	57.27	13.1	30	1.100	0.095	2827.44	0.68
120	150	57.27	51.64	13.1	30	1.109	1.104	2827.44	0.74
						Average for 5 readings = 0.75			



A as a function of D and W.
Redrawn from LUTHIN & KIRKHAM (1949)
Revised by USBR

FIGURE 5. Luthin and Kirkham method (modified from United States Bureau of Reclamation, 1978, p. 71)

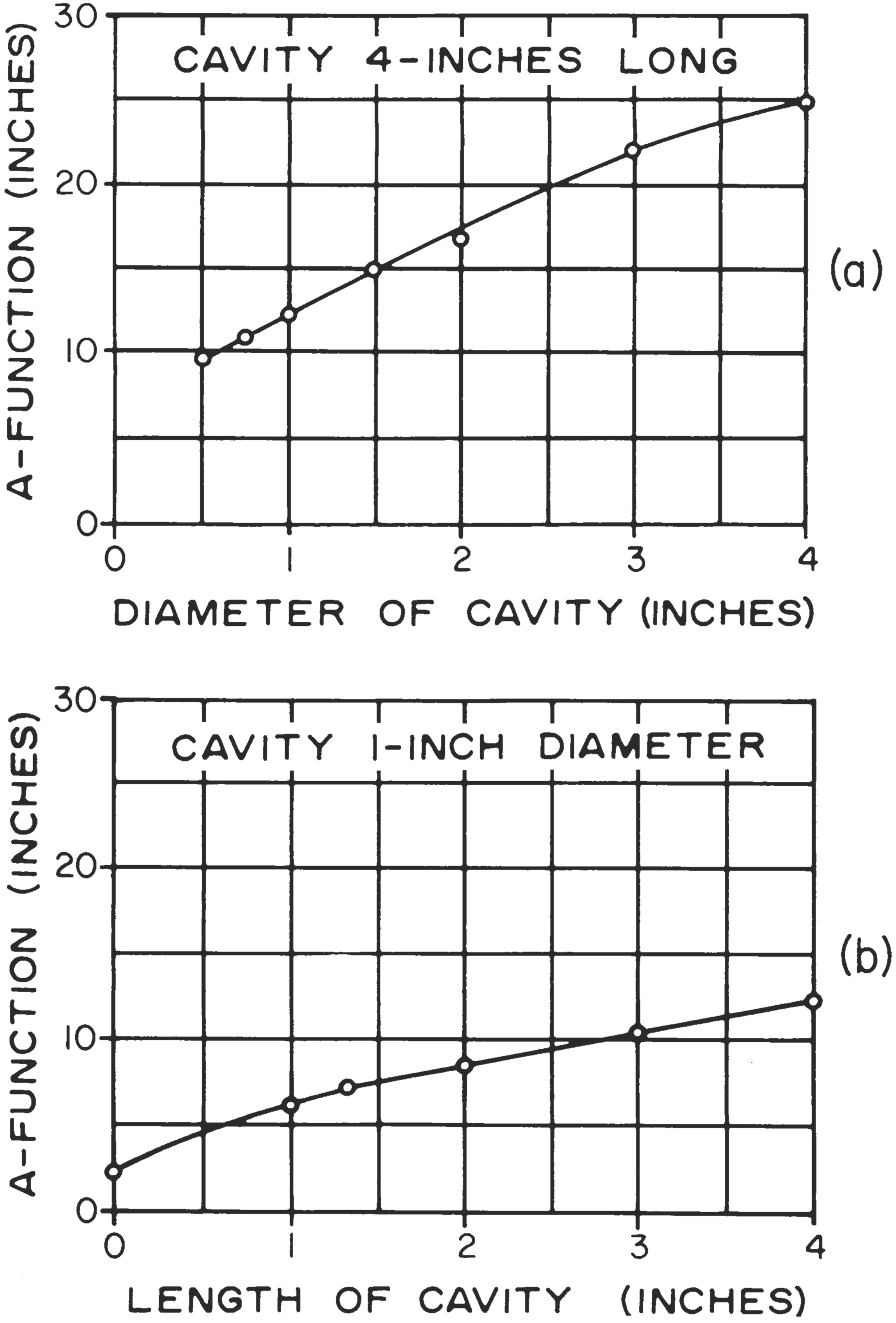


FIGURE 6. Variation of A-function. (a) with intake diameter; (b) with intake length (Luthin and Kirkham, 1949, pp. 353, 354)

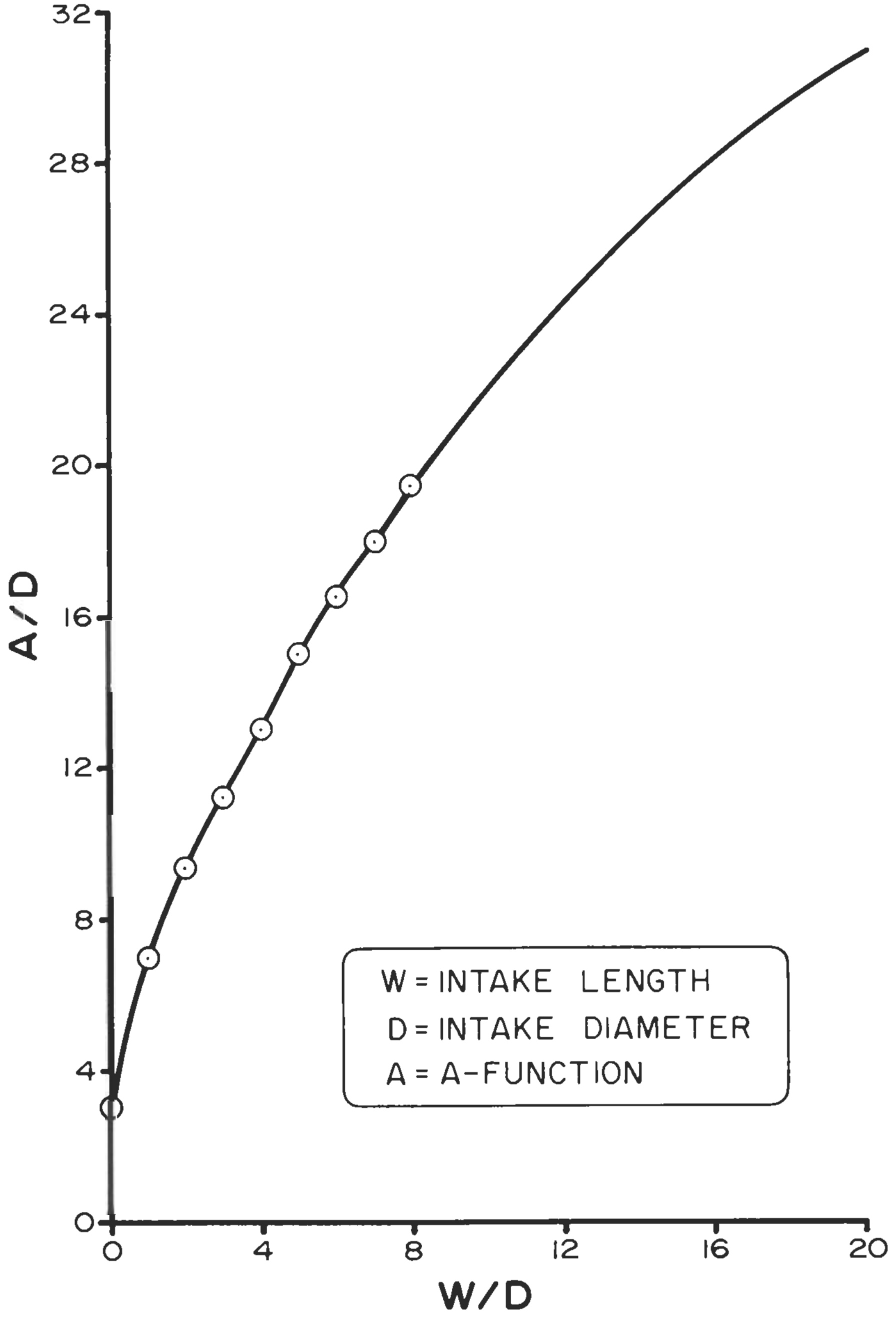


FIGURE 7. Graph of A/D vs. W/D

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0 DEGREE COEFFICIENT= 3.098019   4 DEGREE COEFFICIENT=-8.898026E-03
1 DEGREE COEFFICIENT= 4.268647   5 DEGREE COEFFICIENT= 3.471067E-04
2 DEGREE COEFFICIENT=-.7621251   6 DEGREE COEFFICIENT=-5.245298E-06
3 DEGREE COEFFICIENT= .1119961


FITTED EQUATION IS
Y = A + B*X + C*(X^2) + D*(X^3) + E*(X^4) + F*(X^5) + G*(X^6)

#    X VALUE          Y VALUE          Y CALC          %DEV
 1    0                3                3.098019        +3.16
 2    1                7                6.707981        -4.35
 3    2                9.3              9.351185        +0.55
 4    3                11.2             11.42851        +2.00
 5    4                13               13.20242        +1.53
 6    5                15               14.82913        -1.15
 7    6                16.5             16.3871         -0.69
 8    7                18               17.90165        -0.55
 9    8                19.5             19.36586        -0.69
 10   9                20.7             20.75768        +0.28
 11   10               21.9             22.05323        +0.69
 12   11               23.1             23.23637        +0.59
 13   12               24.3             24.3045         +0.02
 14   13               25.3             25.27051        -0.12
 15   14               26.2             26.16104        -0.15
 16   15               27.1             27.0109         -0.33
 17   16               27.9             27.85376        -0.17
 18   17               28.7             28.70901        +0.03
 19   18               29.5             29.56492        +0.22
 20   19               30.3             30.35795        +0.19
 21   20               31               30.94829        -0.17

COEFFICIENT OF DETERMINATION =  .9998276
COEFFICIENT OF CORRELATION =  .9999138
STANDARD ERROR OF ESTIMATE =  .1531842
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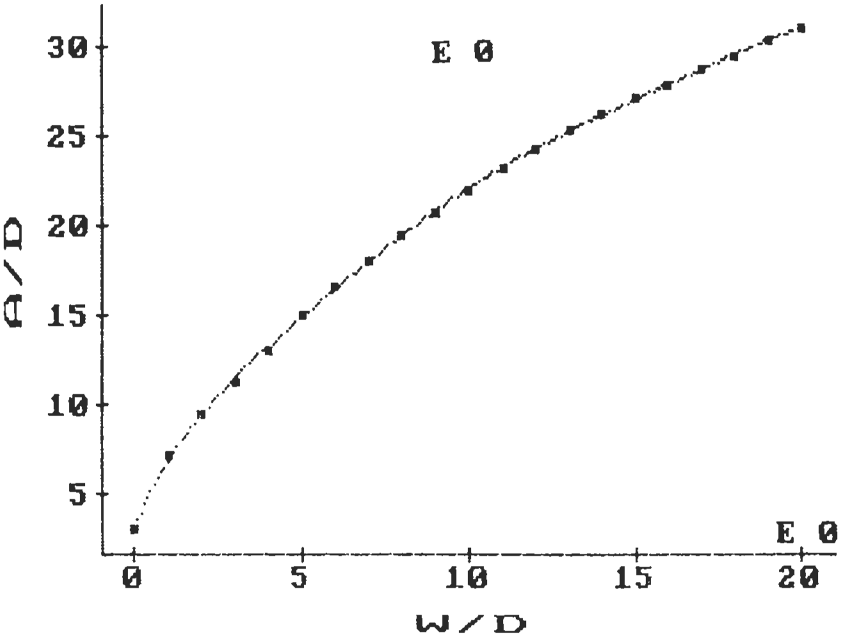
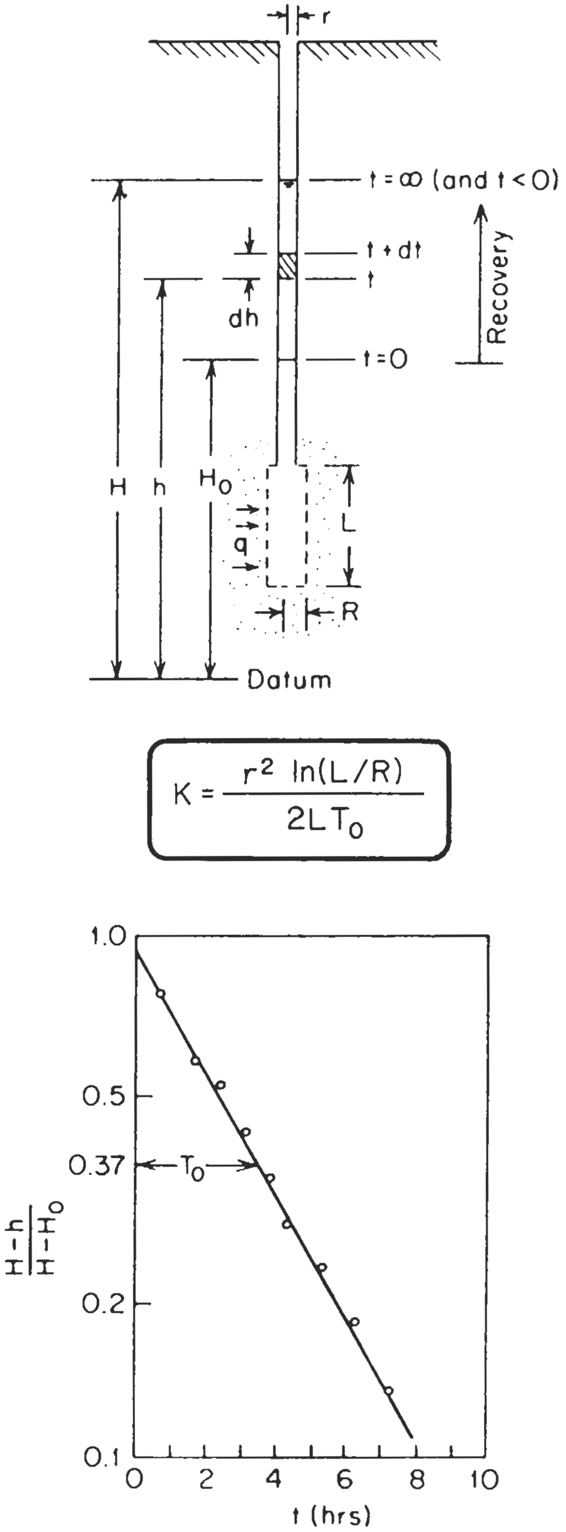


FIGURE 8. Curve analysis of A/D vs. W/D



$$\boxed{K = \frac{r^2 \ln(L/R)}{2LT_0}}$$

FIGURE 9. Hvorslev method (Freeze and Cherry, 1979, p. 340)

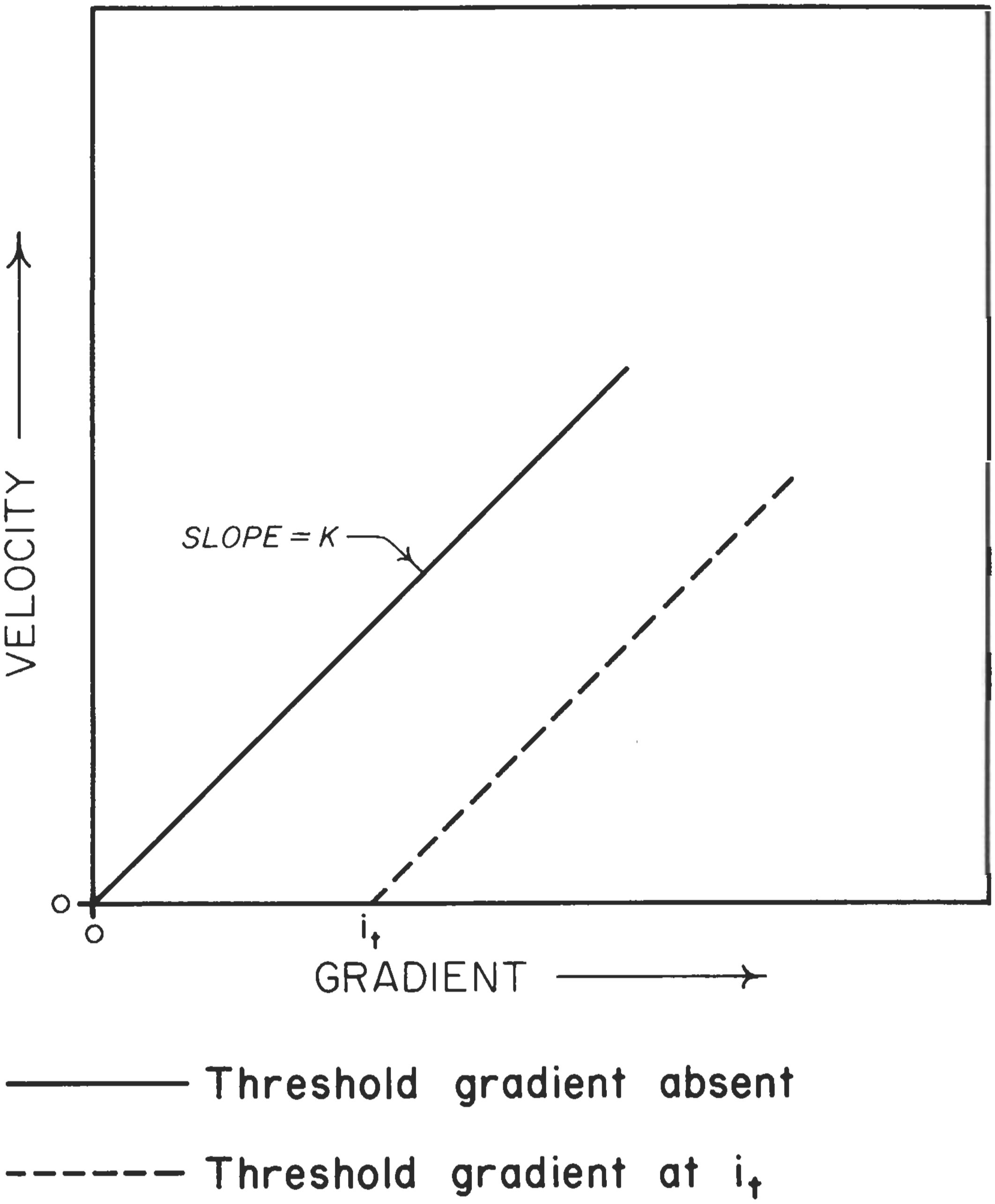


FIGURE 10. Graphical form of Darcy's law